Numerical investigation of the progressive collapse of steel structures due to plan irregularities

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NUMERICAL INVESTIGATION OF THE PROGRESSIVE COLLAPSE OF STEEL STRUCTURES DUE TO PLAN IRREGULARITIES

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Abstract. *Three buildings of different height, regular and irregular as per their plan layout, are designed according to AISC [1] and ASCE7 [2]. These structures were considered to be located in regions with different seismic activity with the purpose of observing their dynamic response under seven load-column-removal scenarios by using non-linear static and dynamic analyses. Non-linear dynamic analyses examine the effect of columns removal on adjacent columns, including node-displacement configurations whilst non-linear static analyses focus on the push-over curve and yield load factor. The results indicate that irregular structures designed in site class C seismic zone do collapse in most of the column removal scenarios. It is also demonstrated that higher level of redundancy showed by the 5-storey with respect to the 2-storey building plays an important role in the prediction of progressive collapse. The collected data lead to various reflections related to regular and irregular building performance under seismic load and the importance of prioritising redundancy and robustness in the context of ultimate limit strength design approaches.*

1 INTRODUCTION

Over the past years, the incidence of blast load in and around buildings and the subsequent progressive collapse have accounted for significant human casualties and structural damage (American Society of Civil Engineers[ASCE]) [3]. Risks and unusual loads potentially causing failure include plane crash, incorrect design or construction, gas explosion, fire, occasional overload, vehicle impact and explosions (National Institute of Standard and Technology[NIST]) [4]. Yet, as the risk relating to such occurrences is not high, buildings are not designed to resist the unusual overload, and neither is the influence of such loads on structures thoroughly examined; thus, structures remain vulnerable to different degrees of damage. Nonetheless, measures exist to mitigate effect of progressive collapse. Such measures are proposed by Facilities Criteria (UFC) and the General Services Administration (GSA) [5,6], both of which address the Alternative Path Method (APM), which remains the most widely practiced measure in fighting progressive collapse.

The proper parameterisation of the procedure is, nevertheless, still being examined. Powel [7], for instance, made a comparison of linear static, non-linear static and non-linear dynamic analyses, finding that using a load factor of 2 in static analyses, we can produce significantly conservative results. Ruth, Marchand, and Williamson [8], likewise, analysed 2D and 3D steel frames so as to demonstrate that a load factor of 2 in non-linear static analyses could be conservative. A factor of ~1.5 was then found to be more accurate in capturing dynamic impact obtained from quasi-static analyses, and a load factor of 2 was found to be more suitable for high-ductility structures, on condition that the materials' behaviour is not elastic-perfectly plastic and that the materials harden over a wide range of strains after yielding. Thus, the authors of the present study recommended using load factors of 2 and 1.5 for ductile structures and others, respectively.

The last decades have also seen wide research into the assessment of the sensitivity, or the lack thereof, to local damage. Gerasimidis and Baniotopoulos [9], studied the disproportionate collapse in steel moment frames and made a comparison of the APM with a numerical approximation, which was based upon β-Newmark and linear Hilbert-Hughes-Taylor procedures. Gerasimidis, Bisbos, and Baniotopoulos [10], reported a parametric study, in which irregular steel frames subject to vertical geometric irregularity had been examined; meanwhile, Gerasimidis, Bisbos, and Baniotopoulos [11], considered structural sensitivity to local damage and introduced the idea of partial damage to structural elements.

Khandelwal, El-Tawil, and Sadek [12], studied the lateral stability of structures by analysing the progressive collapse of steel-braced frames designed seismically, using explicit transient dynamic simulations. In this study, the APM on previously designed 10-storey prototype buildings was used, which demonstrated that concentrically braced frames are far more susceptible to progressive collapse than are their eccentrically braced counterparts. Chen, Peng, Ma, and He [13], studied the effectiveness of horizontal bracing on a steel moment resisting frame, and found that rotation angles and displacements in the model with bracings were nowhere as large as those in the absence of bracing. Kim and Park [14], investigated the progressive collapse-resisting capacity of special truss moment frames in various arbitrary column-removal scenarios. Structures designed according to the AISC seismic provision, it was noted, collapsed, upon the sudden removal of a column, as a result of plastic hinge formation at highly stressed regions. Gerasimidis and Baniotopoulos [15], too, investigated the effect of different strengthening techniques to mitigate progressive collapse in 2D steel moment frames, while Gerasimidis, Deodatis, Kontoroupi, and Ettouney [16], in accordance with the APM, analysed the progressive collapse of a tall steel frame upon the removal of a corner column. Homaioon Ebrahimi, Martinez-Vazquez, and Baniotopoulos [17], investigated the effect of plan irregularities on the progressive collapse of four steel structures located in regions with different seismic activity, and when comparing regular and irregular structures designed in site class E seismic zone, the demand force to capacity ratio (D/C) of the columns in the irregular structures is on average between 1.5 and 2 times that of the regular ones. In addition, Homaioon Ebrahimim, Martinez-Vazquez, and Baniotopoulos [18], analysed the effect of plan irregularities on the progressive collapse of braced and un-braced steel structures located in regions with different seismic activity.The present paper builds on previous research and focus on the impact of three buildings of different height, regular and irregular as per their plan layout, and structural stability evaluated at two distinct seismic regions, C and E, which creates risk scenarios that have not received adequate attention from scholars. Consequently, the spread of damage induced by various column-removal scenarios on three building regular and irregular prototypes is examined and discussed throughout.

2 MODEL STRUCTURES

Three 2, 3 and 5-storey steel structures with regular and irregular plan were selected for the present investigation. Intermediate Steel Moment Frames were pre-designed with ETABS software according to the AISC (2010) and ASCE (2010) to study progressive collapse scenarios in structures. Each of the 6 structures has 3 m height and 6 bays of 4 m wide each, and plan of structures for regular and irregular buildings shown in Figure 1. All structures are assumed to be located in site class C and E seismic zones. The buildings were loaded with 192 kg/m^2 and 520 kg/m^2 dead- and live-load respectively. Further details of the six structures are provided in Table 1 whilst sections of structural members for the regular and irregular structures are given in Tables 2,3,4, and 5 respectively.

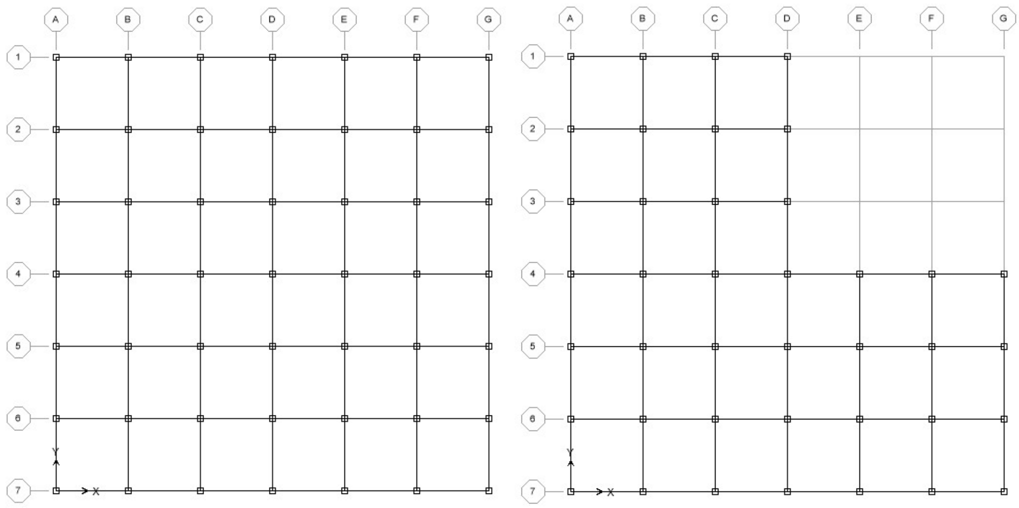


Figure 1: Regular and irregular structures plan

Structure	Seismic zone	Type of Soil	Regularity	Number
Structure 1	C	very dense soil and soft rock	Irregular	2,3,5
Structure 2	E	soft clay soil	Irregular	2,3,5
Structure 3	C	very dense soil and soft rock	Regular	2,3,5
Structure 4	E	soft clay soil	Regular	2,3,5

Table 1. Analysis model structures

		Regular-Seismic region C		Regular-Seismic region E		Irregular-Seismic region C		Irregular-Seismic region E	
		1^{st}	2^{nd}	1^{st}	2^{nd}	1^{st}	2^{nd}	1^{st}	2^{nd}
Column (Box)	b	200,180	180	200,180	180	200,180	180	200	180
	t	10,8	8	10	8	12,10	8	12,10	10
Beam	b_f	150	140	150	140	150	140	150	150
	t_f	8	8	8	8	8	8	8	8
	b_w	250	250	250	250	250	250	250	250
	t_w	8	6	8	6	8	6	8	8

Table 2. Detail of sections used in 2- story structures

		Regular-Seismic region C			Regular-Seismic region E			Irregular-Seismic region C			Irregular-Seismic region E		
		1^{st}	2^{nd}	3^{rd}	1^{st}	2^{nd}	3^{rd}	1^{st}	2^{nd}	3^{rd}	1^{st}	2^{nd}	3^{rd}
Column (Box)	b	200	180	180	200	180	180	200	200	180	200	200	200
	t	12, 10	8	8	12,10	10	8	12	10	8	15,12	12	10
Beam	b_f	150	140	140	150	140	140	150	150	140	150	150	150
	t_f	8	8	8	10,8	8	8	8	8	8	12,10	8	8
	b	250	250	250	250	250	250	250	250	250	250	250	250
	t	8	6	6	8	6	6	8	8	6	8	8	8

Table 3. Detail of sections used in 3- story structures

				Regular-Seismic region C					Regular-Seismic region E		
		1^{st}	2^{nd}	3^{rd}	4^{th}	5^{rd}	1^{st}	2^{nd}	3^{rd}	4^{th}	5^{rd}
Column (Box)	b	200	200	200	200	200	200	200	200	200	200
	t	12,10	10	10	10	10	15,12,10	15,10	12,10	10	10
Beam	b_f	150	150	150	150	150	150	150	150	150	150
	t_f	8	8	8	8	8	12,10	12,10	10,8	8	8
	b_w	250	250	250	250	250	250	250	250	250	250
	t_w	8	8	8	8	8	8	8	8	8	8

Table 4. Detail of sections used in 5- story structures

				Irregular-Seismic region C					Irregular-Seismic region E		
		1^{st}	2^{nd}	3^{rd}	4^{th}	5^{rd}	1^{st}	2^{nd}	3^{rd}	4^{th}	5^{rd}
Column (Box)	b	200	200	200	200	200	200	200	200	200	200
	t	12	10	10	10	10	20,15	15	12	10	10
Beam	b_f	150	150	150	150	150	150	150	150	150	150
	t_f	8	8	8	8	8	15,12,10	15,12,10	12,10,8	8	8
	b_w	250	250	250	250	250	250	250	250	250	250
	t_w	8	8	8	8	8	8	8	8	8	8

Table 5. Detail of sections used in 5- story structures

3 NUMERICAL MODELING

The 3D model structures were numerically analysed with OpenSees software. Non-linear analyses were run considering a simple bi-linear material model with post-yield stiffness of 2% of the initial stiffness. Non-linear beam-column elements were used for modelling the cross-sectional areas as precisely as possible. The plastification over element length and cross-sections were also considered, whereas large displacements effects were also accounted for by the employment of the co-rotational transformation of the geometric stiffness matrix. The dynamic behaviour caused by sudden column removal was not a factor in the load reversal because, in structures subjected to earthquake loads, using a complicated hysteretic model is unnecessary. The fraction of damping was assumed to be 5% which is usually the case for structures with large deformations.

4 ANALYSIS METHOD FOR PROGRESSIVE COLLAPSE

Following the GSA (2013) guidelines, load combinations including 120% of dead load plus 50% of the total live load were gradually applied within a time frame of 5 s. Then, and in order to account for non-linear dynamic effects, the load was maintained steady for the following 2 s. After the 7 s sequence, when gravity load effects are considered to be fully transferred to the structure, a pre-selected column was suddenly removed from the model and the structural response was examined.

In parallel, non-linear static analyses were performed, following the GSA (2003, 2013) recommendation for using a dynamic amplification factor of ~2. That, in order to reflect a ratio of 2 between the load that is applied to the spans that are adjacent to the removed column with respect to that applied on other spans. In this case, vertical loading is applied by following a step-wise increase until the maximum amplified loads are attained or the structure collapses. This vertical pushover analysis procedure, which is often called the 'pushdown analysis method', accounts for non-linear effects which approximate the non-linear dynamic response whilst providing a reliable estimation of the elastic and failure limits of the subject structure.

Derived from the non-linear static analyses, the effective imposed load plotted against the node displacement of the removed column indicates the capacity of a structure against progressive collapse. If the load value is divided by the standard gravity load, the vertical axes of the pushdown capacity curve are converted into dimensionless load factors, as in Eqn. (1). This standardises the load ratio and makes it easier to establish generic observations. The load factor calculated in this way have thus been used herein as a criterion for assessing structural collapse. Namely, if the load factor corresponding to the displacement causing material yield is higher than 1, the structure can withstand the removal of a column, otherwise the structure will collapse.

$$Load\ Factor = \frac{Load}{Nominal\ gravity\ load} \tag{1}$$

5 ANALYSIS RESULTS

As outlined above, in this investigation, the potential collapse of the structures listed in Table 1 is studied under the scenarios set out in Table 6 and Figure 2. In all cases, the column removed correspond those located in the ground floor, as that induced the most critical conditions concerning structural stability. Additionally, a range of column-removal scenarios have been identified in order to induce meaningful configurations of potential failure. In each of these scenarios, a column is suddenly removed and the response of the structure is examined through non-linear dynamic and static analyses, as described above. The columns selected for removal are shown in Figure 3.

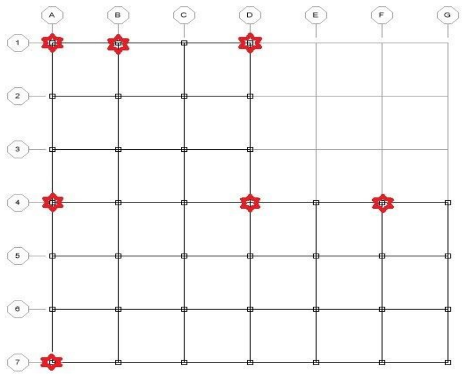


Figure 2: Location of the columns removed for each of the six structures

	Location of removal column			
Number	Storey	Frame	Pier	Scenario notation
1	1	1	A	S1F1PA
2	1	1	B	S1F1PB
3	1	1	D	S1F1PD
4	1	4	A	S1F4PA
5	1	4	D	S1F4PD
6	1	4	F	S1F4PF
7	1	7	A	S1F7PA

Table 6. Column-removal scenarios for each of the six structures.

5.1 Nonlinear static analysis results

5.1.1 Two-storey moment-resisting frame steel structure results

Considering the six regular and irregular structures analysed in this study using nonlinear static analysis. In Fig. 3, a comparison has been conducted between load factors in different column removal scenarios for 2-storey structure.

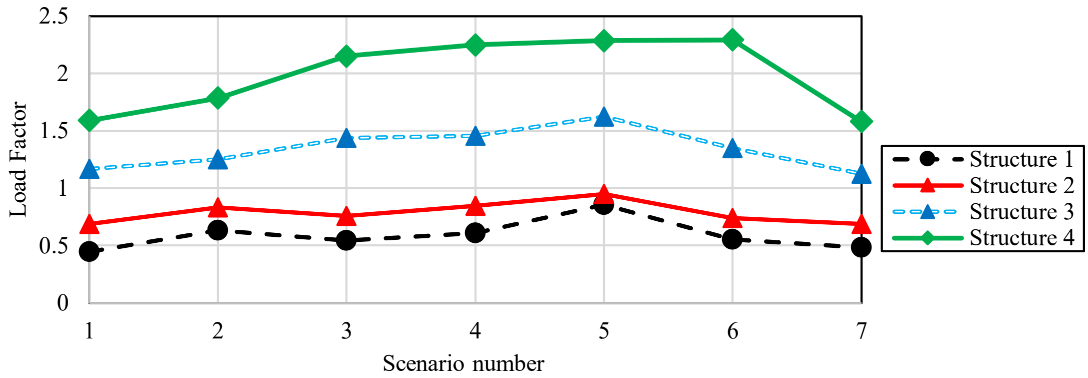


Figure 3: Load factors for all the structures and scenarios in 2-storey

Considering Fig. 3, it may be understood that under all column removal states, the two irregular structures (1 and 2) are not able to bear the force imposed by column removal in two C and E seismic regions, respectively. The most critical state is related to a state when the corner columns on “A” axis are removed. Under various column removal scenarios, both regular structures (3 and 4) managed to bear the load imposed to adjacent columns.

5.1.2 Three-storey moment-resisting frame steel structure results

In Fig. 4, the load factor for regular and irregular structures in 3-storey structure has been shown. As it may be seen, like 2-storey, also here removal of corner column represents maximum damages and minimum structural endurance against progressive collapse are resulted.

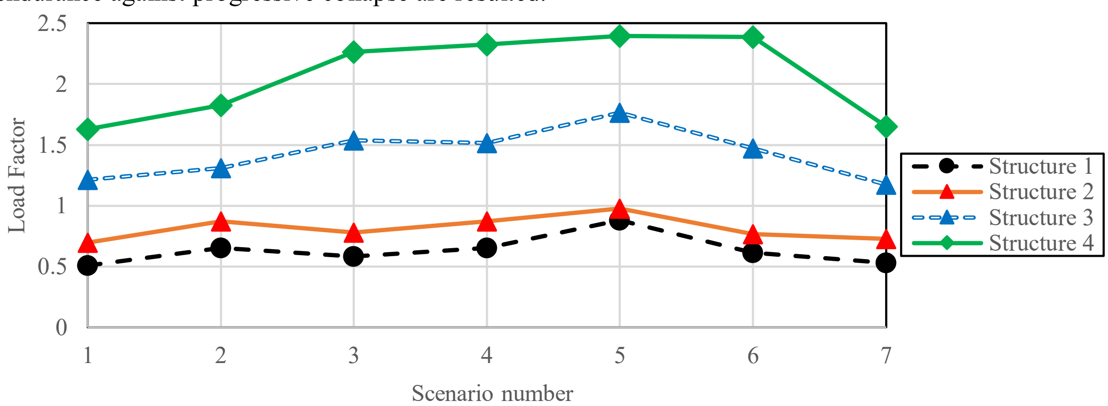


Figure 4: Load factors for all the structures and scenarios in 3-storey

Both regular structures (3 & 4) managed to resist against progressive collapse under various column removal scenarios. Also in these structures, corner (S1F1PA) and internal (S1F4PD) columns removal imposes maximum and minimum damages to the structures, respectively. On the other hand, it is seen that upon increasing structure height from 2 to 3 stories, the structure capacity against progressive collapse also increases. Comparing the load factor in 2- and 3-storey structures with similar status the same issue may be instated. The load factor in structure 1 with scenario 3 is equal to 0.545, while for 3-storey the same is equal to 0.581. In other words, a rough increase of 7% has been observed in the structure capacity.

5.1.3 Five-storey moment-resisting frame steel structure results

Considering Fig. 5, as expected, by increasing the height, and in case of removing S1F4PD Column, the structure may bear the load caused by vertical loading at the removed column. This issue has been expressed using a 1.18 load factor for Structure 2 situated in E seismic region. However, still irregular Structure 1 may not bear the force caused by vertical loading at the removed column and force distribution amongst its adjacent members.

A comparison between the 5-storey structure with those 2- and 3-storey ones demonstrates that under all states the capacity and load factor have increased upon increasing the structure height and developing more stiffness along the structure height.

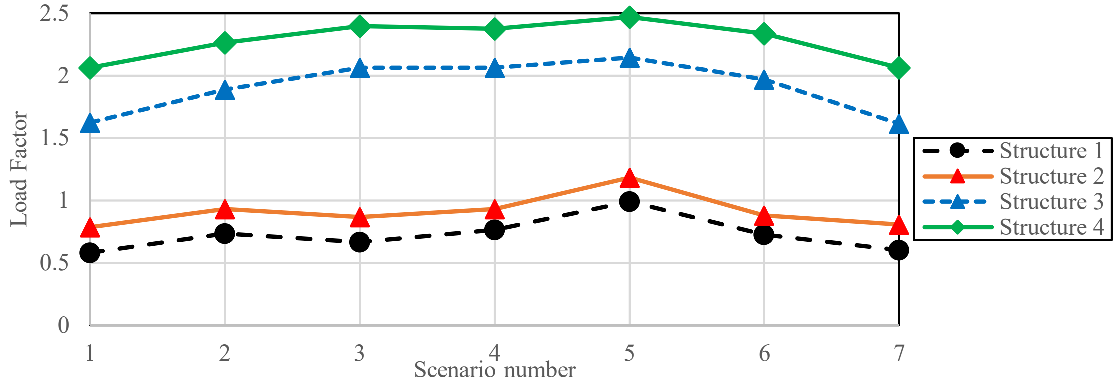


Figure 5: Load factors for all the structures and scenarios in 5-storey

In Fig. 6, the load factor changes in terms of height have been given for all structures (1 through 4). Regarding forces in height ratio changes it may have expressed that the relation between force and height for regular structures is almost linear, while the same is nonlinear for irregular ones.

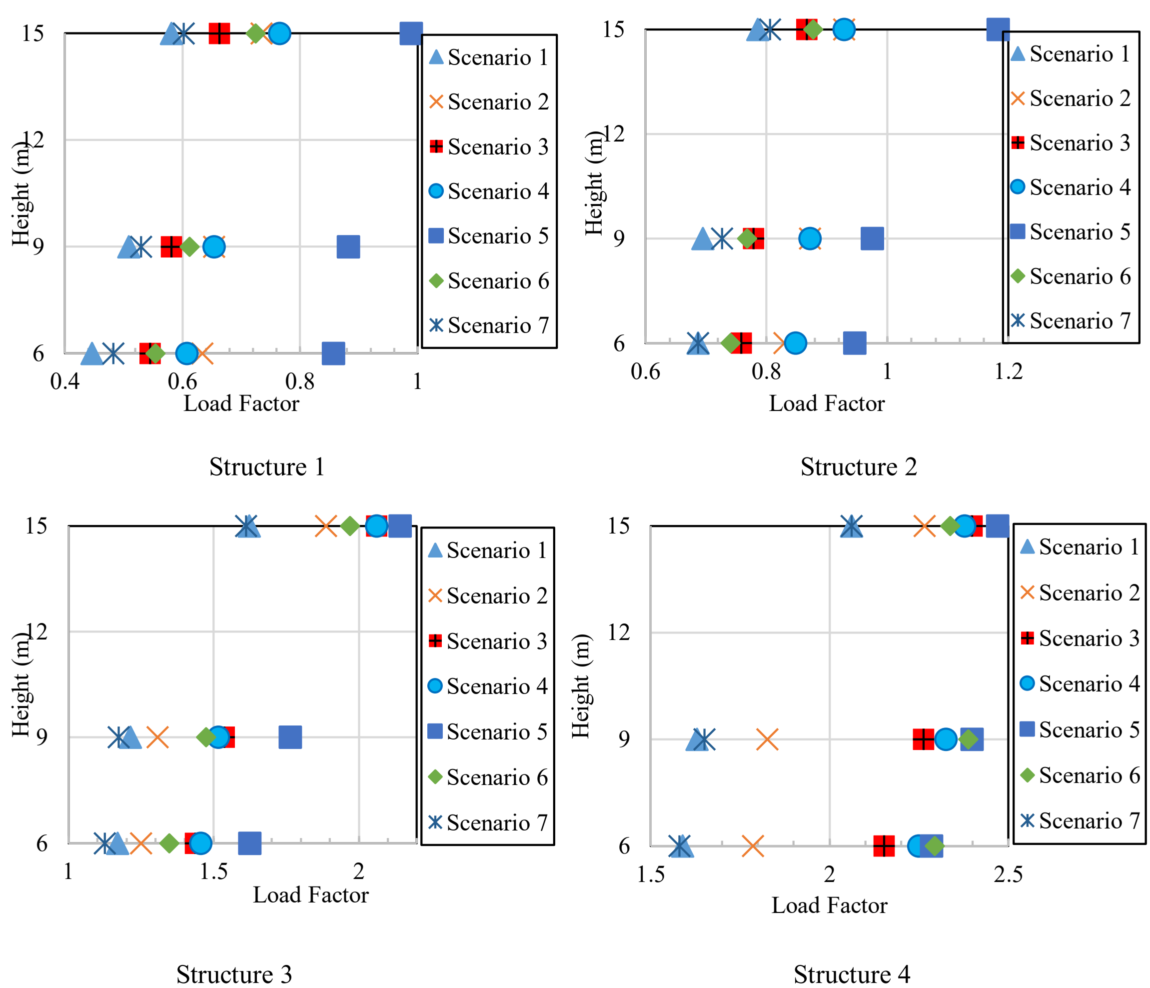
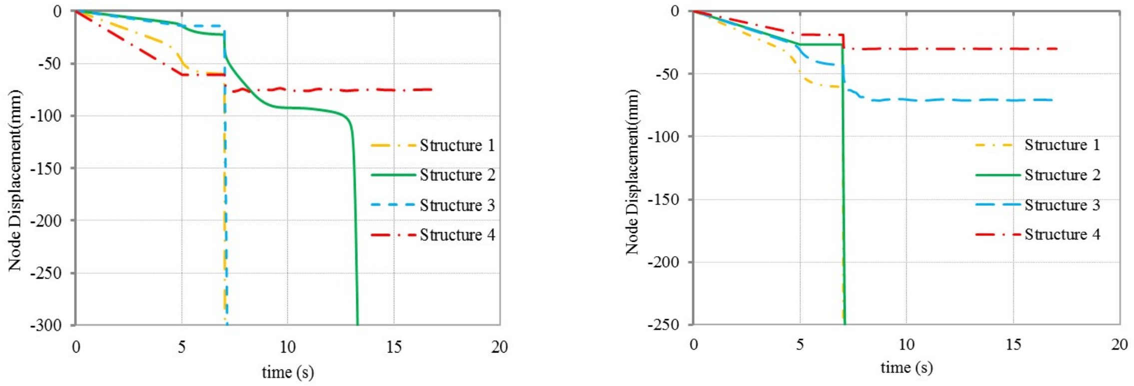


Figure 6: Yield load factors vs. Height for all the structures and scenarios

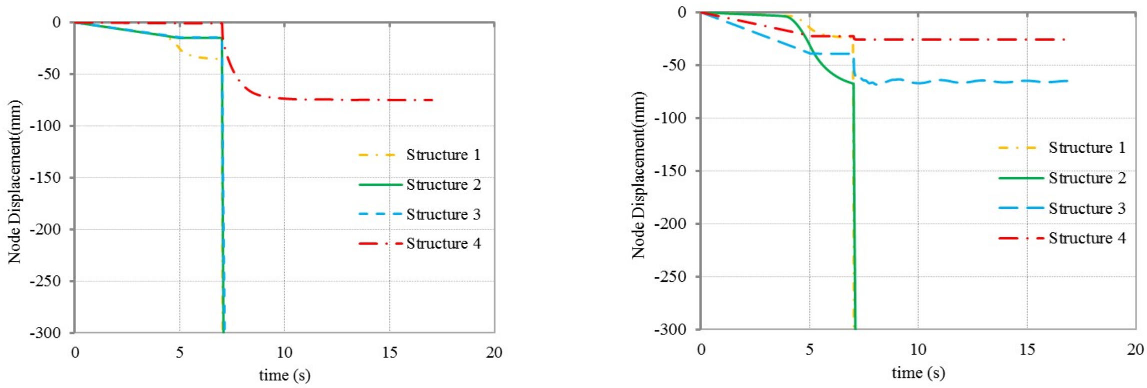
5.2 Nonlinear dynamic analysis results

Non-linear dynamic analyses were used to calculate the peak displacement of the node above a removed column. Figure 7, 8 and 9 shows the results obtained for scenarios 1 and 3 across all structures, respectively. As it can be inferred from the results, node displacements in structures 1 and 2 represent structural collapse of the region around the removed column. In contrast, the node displacement remains constant after 7 s from the removal in all structures 3 and 4 for scenarios 3 which reveals a robust structural performance following the potential failure of the target column.



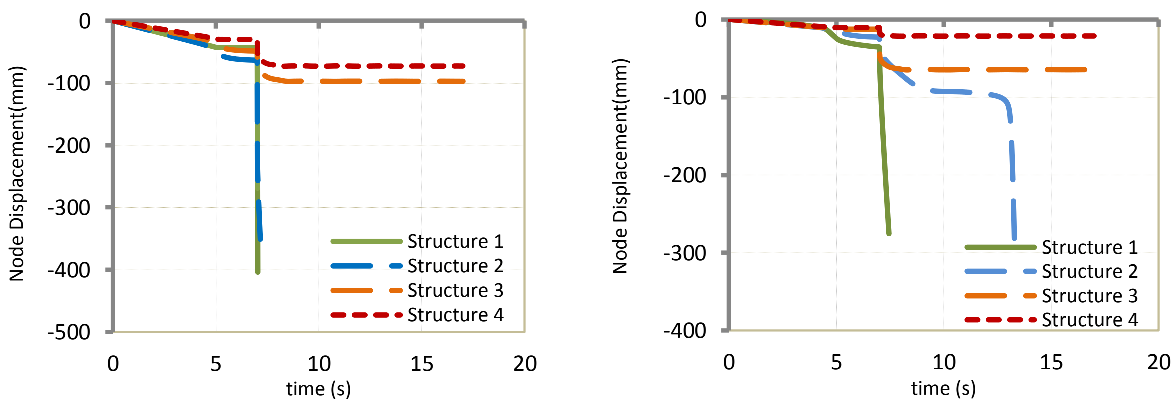
a) Scenario 1 b) Scenario 3

Figure 7: Vertical displacement of removal point of 2-storey structures, a) Scenario 1, b) Scenario 3



a) Scenario 1 b) Scenario 3

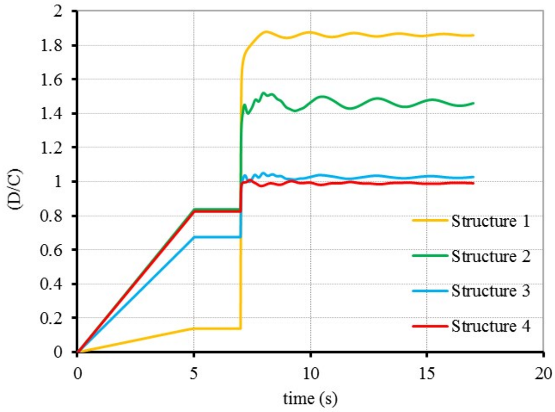
Figure 8: Vertical displacement of removal point of 3-storey structures, a) Scenario 1, b) Scenario 3



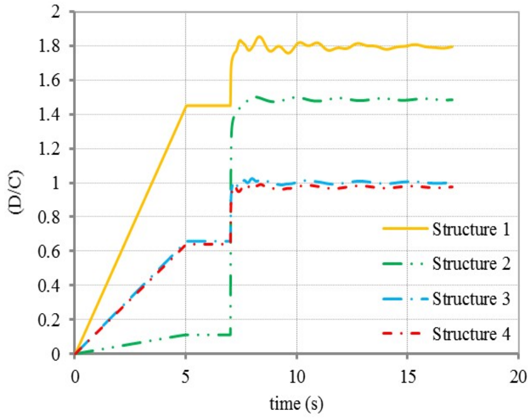
a) Scenario 1 b) Scenario 3

Figure 9: Vertical displacement of removal point of 5-storey structures, a) Scenario 1, b) Scenario 3

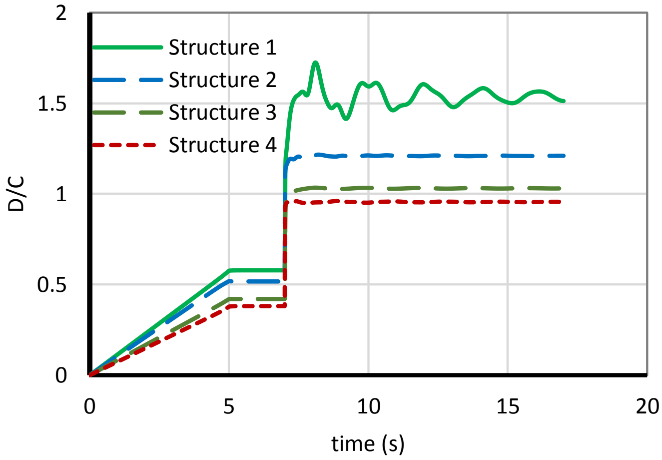
Another key aspect for assessing structural performance under progressive collapse is the force taken by columns that are adjacent to the removed column. In Figure 10 the demand to capacity (D/C) ratio of the columns adjacent to the corner columns in scenario 1 across all the structures is given. It can be seen in this figure that the D/C ratio associated to adjacent columns is around 1 in scenario 1 related to structures 3 and 4. This suggests that columns adjacent to the target-removal one may not be exposed to total damage as alternative load paths do not seem to directly redistribute to those spans.



a) 2-storey structures



b) 3-storey structures



c) 5-storey structures

Figure 10: The demand force to capacity ratio (D/C) of the adjacent columns in Scenario 1
a) 2-storey structures, b) 3-storey structures, c) 5-storey structures

According to the structures 3 and 4 in five storey structures exhibit D/C of 0.6 and 0.73 for scenario 3, respectively. Therefore, these columns are not exposed to collapse under that column-removal scenario, but they would be damaged if belonging to structures 1, 2 and also in these structures the required force for both adjacent columns in the progressive collapse analysis is between 1.2 and 1.8 times the column capacity, which indicates that these columns would have been damaged after the collapse of the target column.
On the other hand, it can be seen the D/C ratio associated to adjacent columns for both two and three storey structures is greater than 1 in scenario 3 related to structures 1,2, and 3 which indicates that these columns would have been damaged after the collapse of the target column. In Table 7, 8, and 9 node displacement and maximum D/C ratio of adjacent columns is given for all the scenarios and structures.

Scenario	Structure 1		Structure 2		Structure 3		Structure 4	
	Node Displacement	D/C	Node Displacement	D/C	Node Displacement	D/C	Node Displacement	D/C
S1F1PA	Fail	1.89	Fail	1.55	Fail	1.16	78	1.01
S1F1PB	Fail	1.42	Fail	1.12	64	0.79	27	0.66
S1F1PD	Fail	1.75	Fail	1.32	71.1	1.82	30	0.72
S1F4PA	Fail	1.33	97	1.09	63.4	0.78	27.1	0.67
S1F4PD	Fail	1.2	95.5	1.07	64.2	0.79	27	0.66
S1F4PF	Fail	1.47	Fail	1.15	71	0.82	28	0.67
S1F7PA	Fail	1.87	Fail	1.52	Fail	1.16	78	1.01

Table 7: Node displacment and maximum D/C ratio of adjacent columns for 2-storey structures

Scenario	Structure 1		Structure 2		Structure 3		Structure 4	
	Node Displacement (mm)	D/C	Node Displacement (mm)	D/C	Node Displacement (mm)	D/C	Node Displacement (mm)	D/C
S1F1PA	Fail	1.85	Fail	1.51	Fail	1.13	75	0.99
S1F1PB	Fail	1.36	95	1.07	62	0.77	24.1	0.62
S1F1PD	Fail	1.68	Fail	1.29	68.2	1.79	26	0.68
S1F4PA	Fail	1.27	93	1.04	61.5	0.77	24.2	0.61
S1F4PD	Fail	1.16	92	1.03	61	0.75	24	0.57
S1F4PF	Fail	1.41	96.5	1.09	68	0.77	25	0.63
S1F7PA	Fail	1.8	Fail	1.45	Fail	1.14	75	0.99

Table 8: Node displacment and maximum D/C ratio of adjacent columns for 3-storey structures

Scenario	Structure 1		Structure 2		Structure 3		Structure 4	
	Node Displacement (mm)	D/C	Node Displacement (mm)	D/C	Node Displacement (mm)	D/C	Node Displacement (mm)	D/C
S1F1PA	Fail	1.77	Fail	1.44	Fail	1.06	42.4	0.96
S1F1PB	Fail	1.29	86	0.92	64.61	0.68	21.4	0.48
S1F1PD	Fail	1.5	Fail	1.23	Fail	1.06	42.4	0.96
S1F4PA	Fail	1.15	84.9	0.87	76.3	0.61	16.3	0.41
S1F4PD	91	1.05	84	0.83	65.7	0.52	14.1	0.39
S1F4PF	Fail	1.31	87.8	0.94	57.6	0.62	18.2	0.42
S1F7PA	Fail	1.67	Fail	1.38	Fail	1.03	40.1	0.91

Table 9: Node displacment and maximum D/C ratio of adjacent columns for 5-storey structures

6 CONCLUSION

In this study, three ,2, 3 and 5-storey irregular and regular steel structures with moment-resisting frame were designed in site class C and E according to the AISC (2010) and ASCE7 (2010). The effect of plan irregularities and type of seismic regionalisation on progressive collapse have been analysed under various column removal scenarios. The results of the analyses reveal that in cases where the structural plans were similar, the structure designed in a region in site class E seismic risk has less collapse potential. Moreover, the potential for progressive collapse was identified to be higher for buildings with plan irregularities and or in site class C. It was also seen that the displacement of the node above the removed column and the D/C ratio of the columns adjacent to the one removed could provide a fair indication of the risk of overall collapse of structures.
The results showed that upon increasing structure height from 2 to 3 stories, the structure capacity against progressive collapse also increases. Comparing the yield load factor in two 2- and 3-storey structures with similar status the same issue may be instated. The yield load factor in structure 1 with scenario 3 was equal to 0.545, whilst for 3-storey the same was equal to 0.581. In other words, a rough increase of 7% has been observed in the structure capacity. Generally, a comparison between the 5-storey structure with those 2- and 3-storey ones demonstrated that under all states the capacity and yield load factor have increased upon increasing the structure height.

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